DESIGN MEMORANDUM

FOR THE

MONTANO PUMP STATION

FOR

THE CITY OF ALBUQUERQUE, NEW MEXICO

Prepared By:

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(85-517)

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SECTION 1

EXECUTIVE SUMMARY

The Montano Bridge project involves the construction of a new roadway between Coors Boulevard and Rio Grande Boulevard with a 2,000 ft. long bridge over the Rio Grande, and widening and realignment of Montano Road from 2nd Street to Rio Grande Boulevard. The project also includes the construction of a 106 ft. long bridge on Rio Grande Boulevard over Montano Road.

The construction of the overpass bridge on Rio Grande Boulevard will create a sump near that crossing. This depression, coupled with the collection of runoff from lands paralleling Montano Road east, provides the need for a pump station near the Montano/Rio Grande Intersection (see Figure 1 - Pump Station Site).

The design of the pump station is based on a maximum inflow of 95 cfs (42,600 gpm). A collector line, 60 in. to 66 in. RCP, will extend from 2nd Street west to the pump station at Rio Grande Boulevard. Second Street will be connected to an existing storm sewer that extends from I-25. The collector line will have a siphon under the Alameda Drain. In addition there will be a 30 in. storm drain from the depression at the intersection of Montano Road and Rio Grande Boulevard to the pump station (see Figure 1).

Four 10,800 gpm submersible pumps have been selected to discharge the design flow of 95 cfs. The design includes an additional pump to serve as a backup in the event one pump is taken out of service for maintenance or repair. All pumps will be connected in parallel so each individual pump meets the demands of the total system head, and their combined pumping action provides the necessary relief based on the requirements of the incoming flow. Two smaller 500 gpm submersible pumps shall be provided as a means of discharging the nuisance flows and draining the wet well.

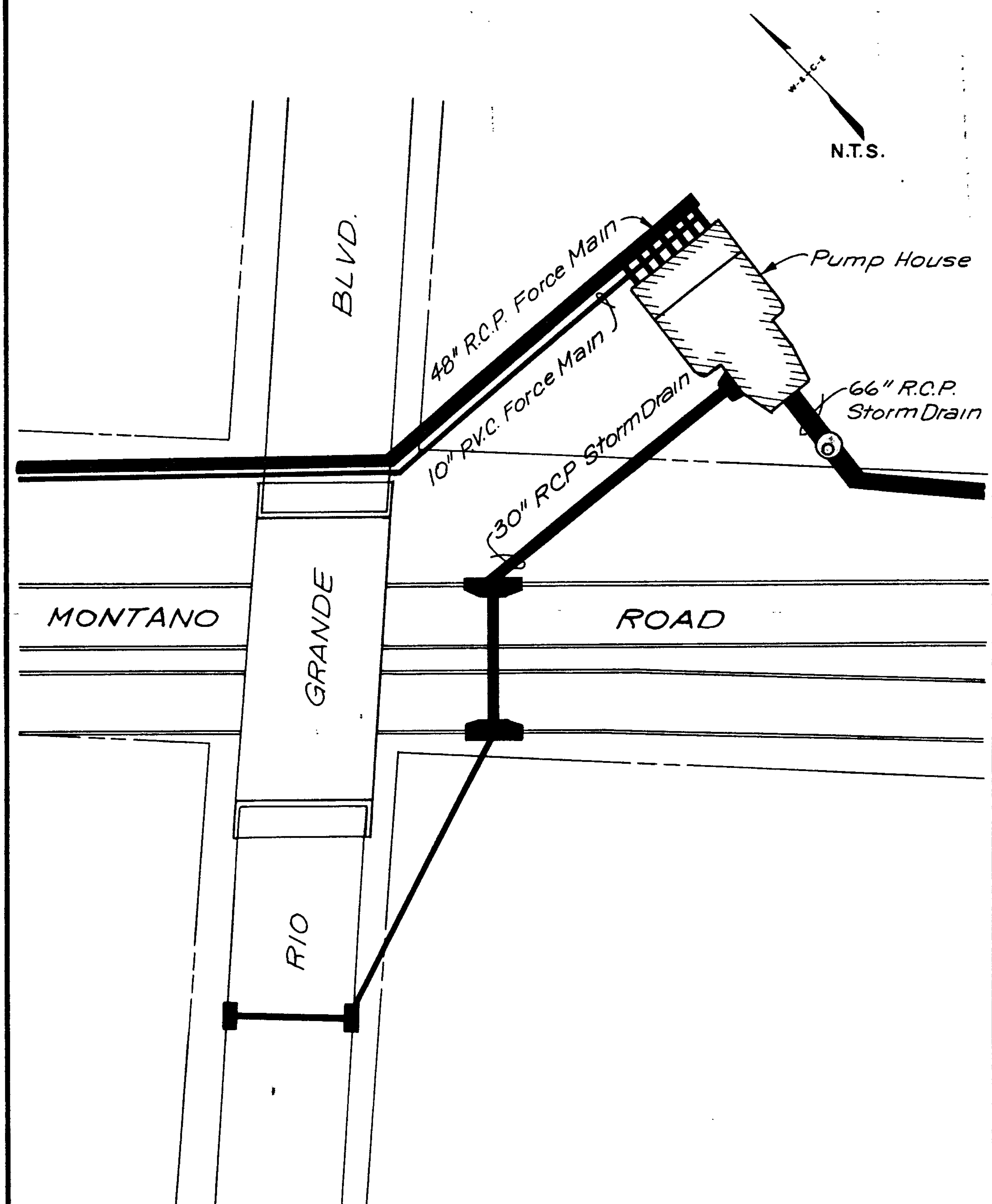
Check and gate valves the size of each discharge shall be installed on line with each pump.

In the event of a power failure during the operation of any or all of the pumps, surge relief shall be provided to dissipate transients.

A mechanical barscreen (rack spacing 2-1/2 in.) will be centrally located in the inlet channel. Screening deposits shall be lifted and discharged into a ground level storage container. Because a portion of the debris carried by the flow will not be screened out, the pumps shall be able to pass 3 in. solids.

Two force mains, each approximately 2,900 L.F., will extend from the pump station site to the river. The last 150 L.F. is considered gravity line. A 48 in. RCP and a 10 in. PVC main will discharge storm water runoff into a riprap-lined channel for conveyance to the river.

Access to and around the site shall accommodate an attenuated WB-40 vehicle. Security fencing and ample lighting for maintenance will also be provided onsite.



WILSON & COMPANY ENGINEERS & ARCHITECTS	FIGURE 1 PUMP STATION SITE				
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The Montano River Crossing consists of roadwork from Rio Grande Boulevard to Coors, construction of the river bridge and the Rio Grande/Montano grade separation, and exterior site work and landscaping at the pump station.

The roadway improvement portion of the design involves roadwork from 2nd Street to Rio Grande Boulevard, complete installation of the storm drain system, and construction of the pump station and related facilities.

SECTION 2

DESIGN ANALYSIS

INTRODUCTION

The purpose of the Montano Pump Station is to collect storm water from the North Valley between Rio Grande Boulevard and I-25 and transmit it through two force mains to the river.

BASIS OF DESIGN

An analysis was made of the immediate drainage area to determine the runoff flow. This flow includes runoff from this project plus runoff contributed from an area east of 2nd Street. The anticipated peak flow to the pump station (shown on the Hydrograph - Figure 2) from a 100-year storm (which in any given year has a probable frequency of occurring once in 100 years) was calculated to be 95 cfs. As shown on the Hydrograph, flows from storms in the two described drainage areas will not peak simutaneously; consequently the flows from these two areas are not additive. The maximum peak flow, 95 cfs, was used as the design flow for the storm drainage system and pump station. Flow in excess of 95 cfs will be directed to the Alameda Drain by means of a control device.

HYDRAULIC DESIGN

General

The sizing of inlet and wet well structures was based on two parameters: design flow and maximum wet well elevation. The design flow to be carried by the storm drain system will be 95 cfs. Given this capacity requirement and the limited grade available, it will be necessary to set the maximum water level elevation in the wet well at 4958.0 ft. (see Figure 3 - Pump Station Profile). This elevation will provide the required head to insure capacity for the design flow; anything greater would cause a decrease in flow and the occurrence of surcharged inlets and flooded intersections.

INLET STRUCTURE

Inlet Channel

The inlet channel as shown in Figure 3 widens out symmetrically from 5-1/2 ft. at the entrance to 10 ft. at mid-length. The floor drops 1-1/2 ft. over a total inlet channel length of 28 ft.

The cross section in front of the barscreen will be slightly V-shaped to funnel low flows and debris toward the center of the channel. There will be a 2 ft. long transition both into and out of this cross section.

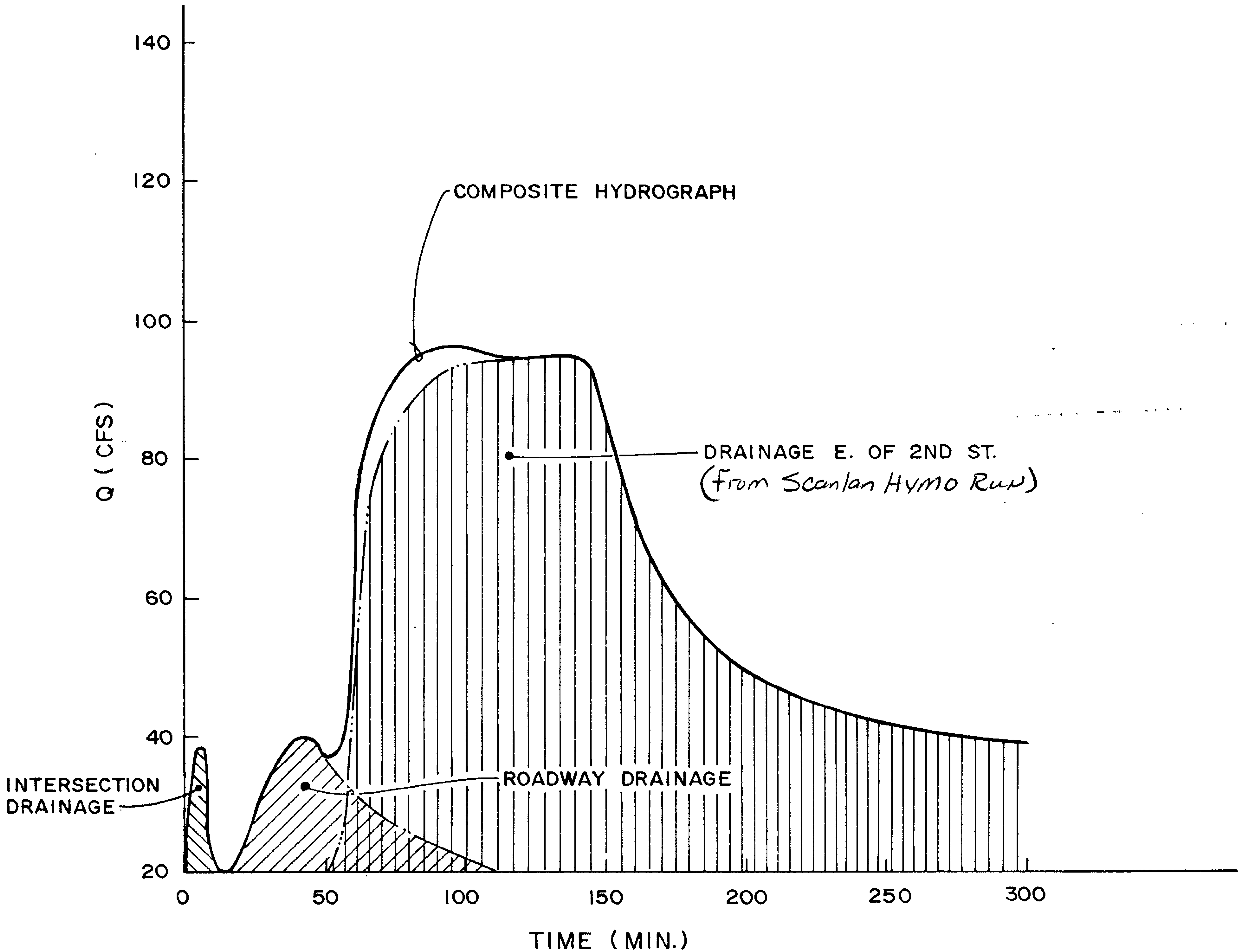

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FIGURE 2

HYDROGRAPH AT PUMP STATION-100 YR. STORM

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Overflow channels are provided as an alternative flow route if the barscreen becomes clogged. The floor of the overflow channel parallels the inlet channel. The overflow channel invert elevation, 4,959.80 ft., is equal to the maximum water elevation in the storm drain.

Barscreen

A barscreen will be required in the inlet channel to remove any large debris carried in the flow. It will be located 16 ft. downstream of the 66 in. storm drain outlet and will have an approximate maximum depth of flow of 3 ft. The width of the barscreen, determined by the width of the channel, shall be 10 ft. Given a bar thickness of 3/8 in. and a bar rack opening of 2-1/2 in., the area used to calculate velocity through the barscreen is 26 ft^2. Based on the design flow of 95 cfs, the resulting calculated velocity will be 3.64 fps. The maximum recommended velocity through the barscreen is 4.5 fps.

A mechanical rake will be activated by direct operation of the pumps. An alarm system will be installed and set to go off in the event the barscreen clogs or the rake becomes immovable.

Wet Well

Flow through the inlet channel will discharge against baffles set 5 ft. downstream from the entrance of the wet well. The outfall drop from the inlet channel to the wet well will be approximately 7-1/2 ft. Because the maximum water level elevation in the wet well may not exceed 4958.0 ft., adequate depth (for pump cycling) requires the main floor elevation be set at 4946.0 (see Figure 3). The main portion of the floor will slope downward concentrically 1/2 ft. toward the opening of the sump pit. This will accomplish two things: it will keep the wet well floor relatively free of grit and debris that passes the barscreen and it will provide easy drainage during wet well cleaning.

Sump Pit

The sump pit will be centrally located along the back wall of the wet well. It will contain two 20 hp pumps and its overall size will be maximized to avoid unnecessary cycling of the pumps. The sump pit floor will begin sloping 3 ft. from the centerline of the baffles. It will drop 3 ft. in 10 ft. and level off for a distance of 7 ft. The sump pit shall be 10 ft. wide and the adjacent large pumps shall be located 2.75 ft. from each side.

Pumps

Five large pumps will be located approximately 8 ft. off the back wall and 12 ft. downstream of the centerline of baffles. There will be two to the north and three to the south of the sump pit.

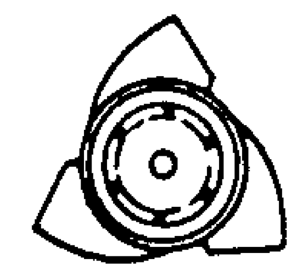
Selection of the pumps was made on the basis of maximum design flow, moderately high efficiency and low Net Positive Suction Head (NPSH). Five submersible 214 hp pumps were selected; four main and one standby. The two 20 hp pumps will be installed for draining the wet well and nuisance flow handling.

PERFORMANCE CURVE

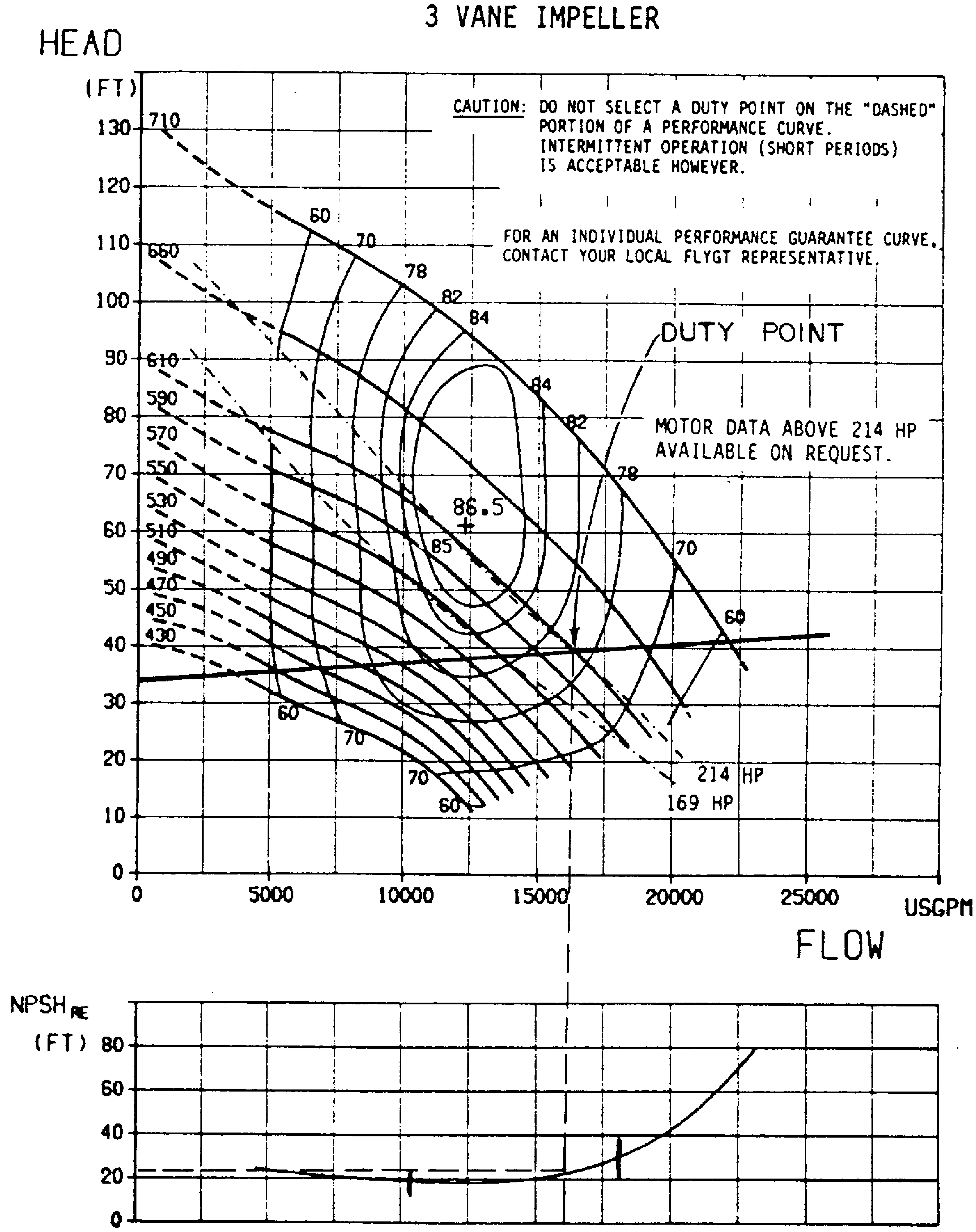
SECTION	PAGE
3530	8/1030
SUPERSEDES	ISSUED
	4/86

CP/CT-3530

DETERMINE THE OPERATING SPAN ON THE PUMP CURVE SELECTED. ALL PUMP CURVES OR PORTIONS OF PUMP CURVES BELOW THE 169 HP CURVE(Dotted Lines)REQUIRE THE 169 HP MOTOR. ALL PUMP CURVES OR PORTIONS OF PUMP CURVES ABOVE THE 169 HP CURVE AND BELOW THE 214 HP CURVE REQUIRE THE 214 HP MOTOR ETC.

169;214 HP - 710 RPM 3Ø: 460,575V	WASTEWATER IMPELLER 1030	

3 VANE IMPELLER



— FIGURE 4 —

PERFORMANCE CURVES ARE BASED ON TESTS WITH CLEAR WATER AT AMBIENT TEMPERATURE.

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The design requires parallel pump connections and separate force mains for the two different sized pumps. The 20 hp pumps are likely to handle most of the debris and grit; therefore, in order to keep velocities sufficient to prevent solids from settling out, the smaller pumps will discharge into a 10 in. PVC force main. A 48 in. force main will collect the flows from the larger 214 hp pumps.

Design Calculations (Large Pumps)

Total Dynamic Head (TDH)

Given: Q = 95 cfs
*Minimum water elevation in wet well = 4,950.5 ft.
Top of pipe at discharge of force main = 4,985 ft.
Length of force main = 2,750 ft.
Slope of the gravity line = .033 ft./ft.

Equations: $TDH = H_{lift} + H_{velocity} + H_{friction}$

$h_f = 4.727\ l\ Q^{1.85}/C^{1.85}D^{4.87}$ (Hazen Williams Eqtn.)

l_{20} = 31.5 ft.; $l_{f.m.}$ = 2,750 ft.

l_{20} has a 20 in. discharge; l_{fm} has a 48 in. discharge

$Q = \frac{1.49}{n} AR_h^{2/3}\ S_o^{1/2}$ (Chezy-Manning Eqtn.)

$KV^2/2g$ = headloss through fittings/valves

Assume: C = 110

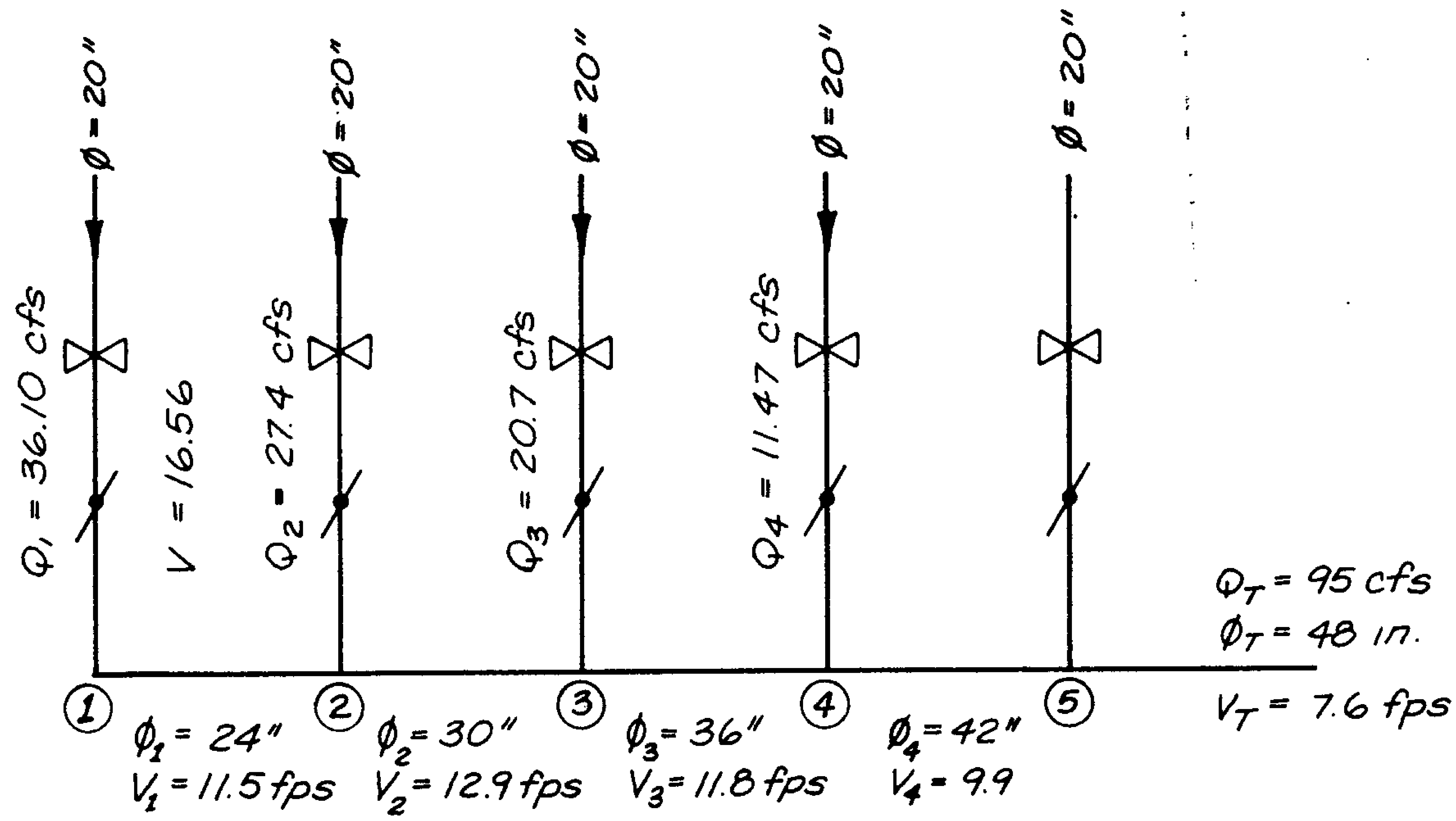
Solution: Worst case scenario looks at the end pump coming on and the losses accumulated as flows travel the line. (See Schematic of Valve Sequence - Figure 6.)

A. H_{lift}: H_l = 4,985 - 4,950.5 = 34.5 ft.

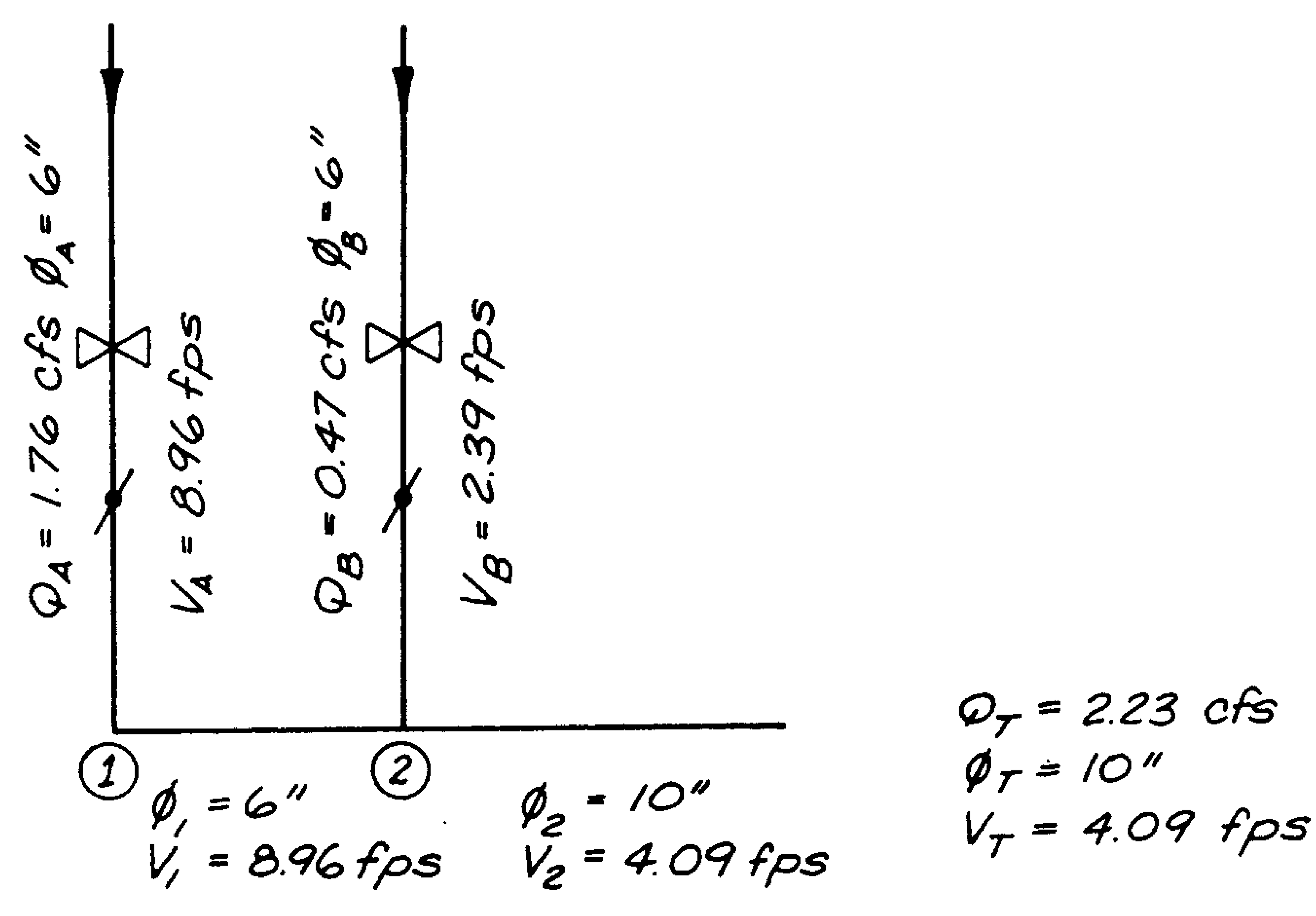
B. $H_{velocity}$: Velocity through 48 in. pipe $= \frac{Q}{A} = \frac{95\ \text{cfs}}{12.57\ \text{ft}^2} = 7.56$ fps

Q through the 20 in.: Originally it is a guess, but ultimately the duty point is read directly off the graph of the system curve and the pump curve.
Q_{20} = 36.10 cfs

*This was the initial elevation assumed. The minimum NPSH value calculated was 4951.0 which means that the above calculations are conservative.



Note: In distributing the flows it was assumed that maximum flow occurs through the 1st pump with gradually decreasing flows as each successive pump turns on – this, however is not realistic and is only used to create a worst case scenario.



WILSON & COMPANY ENGINEERS ARCHITECTS	FIGURE 6 SCHEMATIC OF VALVE SEQUENCE		
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Velocity through the 20 in. pipe $= \dfrac{36.10 \text{ cfs}}{2.18 \text{ ft}^2} = 16.56$ fps

$$H_v = \frac{V^2}{2g} = \frac{(7.56)^2}{2(32.174)} = \underline{.9 \text{ ft.}}$$

C. $H_{friction}$: H_f = headloss through the valves
+ headloss through the header
+ h_f (see sketch of schematic)

Headloss through the valves (per pump):

Check valve (1): $.6 \dfrac{(16.56)^2}{2g} = 2.56$ ft.

Gate valve (1): $.1 \dfrac{(16.56)^2}{2g} = 0.43$ ft.

20" pipe connections (1):

$$4.727(31.5)36.10^{1.85}/110^{1.85}1.67^{4.87} = 1.56 \text{ ft.}$$

90° elbows (2): $\dfrac{.2(.36)(16.56)^2}{2g} = 3.07$ ft.

Total = 7.62 ft.

Headloss through the header:

$$0.72 \frac{(11.5)^2}{2g} +$$

$$0.66 \frac{(12.9)^2}{2g} + 0.22 \frac{(12.9)^2}{2g} +$$

$$0.66 \frac{(11.8)^2}{2g} + 0.22 \frac{(11.8)^2}{2g} +$$

$$+0.60 \frac{(9.9)^2}{2g} + 2 \left[0.20\frac{(9.9)^2}{2g}\right] = 7.18 \text{ ft.}$$

h_f = headlosses through force main

$$h_f = 4.727(2750)\ 95^{1.85}/110^{1.85}4^{4.87}$$

$$= 11.59 \text{ ft.}$$

$$H_f = 7.62 + 7.18 + 11.59 = \underline{26.39 \text{ ft.}}$$

D. TDH: TDH = 34.50 + 0.90 + 26.39 = $\underline{\underline{61.79 \text{ ft.}}}$

CHECK: Check to see that adequate slope is provided for the gravity line:

S_o prov'd = .033

$$Q = \frac{1.49}{n} AR_h^{2/3} S_o^{1/2} = \frac{1.49}{.013} (12.57) S_o^{1/2} = 95 \text{ cfs}$$

S_o req'd = .0044; S_o req'd < S_o prov'd

System Curve

Refer to Pump Curves - Figure 4 and Figure 5

Given: Q = 95 cfs
Static Lift = 34.5 ft.
TDH = 61.79 ft.

Equation: $H_{p(system)}$ = Static Lift + KQ^2

Solution: $H_{p(system)} = 34.5 + K(95)^2 = 61.79$ ft.

$K = .00302$

$H_{p(system)} = 34.5 + .00302Q^2$

Design Calculations (Small Pumps)

Total Dynamic Head (TDH):

Given: Q = 1,000 gpm = 2.23 cfs

Elevation of top of force main at discharge = 4984.83 ft.

*Minimum elevation in wet well = 4944.42 ft.

l_6 = 32 ft.
$l_{f.m.}$ = 2750 ft.

Equations: $TDH = H_{lift} + H_{velocity} + H_{friction}$

$h_f = 4.727 \; l \; Q^{1.85} / C^{1.85} D^{4.87}$ (Hazen Williams Eqtn.)

Assume: Diameter of discharge = 6 in.
C = 110

Solution: (See Figure 6)

A. H_{lift}: H_l = 4984.83 - 4944.42 = 40.41 ft.

*This was the initial elevation assumed. The minimum NPSH value calculated was 4945.0 which means that the above calculations are conservative.

B. $H_{velocity}$: Velocity through 10" pipe = $\frac{Q}{A} = \frac{2.23}{.55} = 4.09$ fps

$H_v = \frac{V^2}{2g} = \underline{0.26\ ft.}$

C. $H_{friction}$: H_f = head loss through valves
+ head loss through tee's
+ H_f (see sketch of schematic)

Velocity through 6" pipe = $\frac{Q}{A} = \frac{1.76}{.20} = 8.96$ fps

Head loss through the valves (per pump):

Check valves (1): $\frac{.68(8.96)^2}{2g}$ = 0.85 ft.

Gate valves (1): $\frac{.12(8.96)^2}{2g}$ = 0.15 ft.

6 in. pipe connections (1):

$4.727(32)(1.76)^{1.85}/110^{1.85}.5^{4.87}$ = 2.11 ft.

90° elbows (2): $\frac{0.45(8.96)^2}{g}$ = 1.12 ft.

Total = 3.11 ft.

Headloss through the tees:

$$\frac{0.90(8.96)^2}{2g} + \frac{0.90(.47)^2}{2g} + \frac{0.3(8.96)^2}{2g} = 1.50 \text{ ft.}$$

h_f:

$h_f = 4.727(2750)\ 2.23^{1.85}/110^{1.85}.83^{4.87} = 23.30$ ft.

$H_f = 23.30 + 1.50 + 3.11 = \underline{27.91\ ft}.$

D. TDH: TDH = 40.41 + 0.26 + 27.91 = $\underline{\underline{68.58\ ft.}}$

System Curve

Refer to Pump Curves - Figures 7 and 8.

Given: Q = 2.23 cfs
Static Lift = 40.41 ft.
TDH = 68.58 ft.

Equation: $H_{p(system)}$ = Static Lift + KQ^2

Solution: $H_{p(system)} = 40.41 + K(2.23)^2 = 68.58$ ft.

$K = 5.6647$

$H_{p(system)} = 40.41 + 5.6647Q^2$

NPSH (Net Positive Suction Head) and Efficiency Considerations. Selection of pumps was made based, by order of importance, on capacity, minimizing NPSH and maximizing efficiency. Four Flygt CP-3530 (Wastewater Impeller 1030-Curve 610) pumps were selected to handle the 95 cfs flow. As indicated on Figures 4 and 5, the following parameters with regard to these pumps are:

- The duty point = 16,500 gpm (36.80 cfs)
- Capacity per pump (taken as total capacity supplied divided by four) = 10,800 gpm (24.1 cfs)
- Efficiency = 80% (approximately)
- Minimum NPSH elevation in the wet well = 4,951.00 ft.
- Required NPSH = 24 ft.
- Available NPSH = 27.4 ft.

Calculations for available NPSH are as follows:

Given: $H_{abso} = 24.0$ ft.

K' = factor used to adjust for 5,000 ft. altitude and storm conditions = 0.85

H_s = elevation of water above center line of pump = 1.0 ft.

$V = 16.56$ fps

$H_{vp} = 1.4$ ft. (based on temperature = 85°)

Equations: $NPSH_{(available)} = H_{abso'} + H_s - H_f - H_{vp}$

$H_{abso'} = K' H_{abso}$

$H_f = K \dfrac{V^2}{2g}$; K for a suction bell = 0.1

Solution: $H_{abso'} = .85(28.2) = 24.0$ ft.

$H_f = 0.1 \dfrac{(16.56)^2}{2\ (32.174)} = .43$ ft.

$NPSH_{(available)} = 24.0 + 1.0 - .43 - 1.4 = \underline{27.4\text{ ft}}.$

A similar method was used to obtain an $NPSH_{(available)}$ for the smaller pump. Please refer to Figure 7 and the data below:

- $NPSH_{(available)}$ = 23.5 ft.
- $NPSH_{(required)}$ = 10 ft.
- Minimum NPSH elevation in the sump pit = 4,945.00 ft.
- Efficiency = 63% (approximately)

Pump Level Settings (For Large Pumps). Calculations were performed to determine the levels in the wet well at which the pumps will cycle on and off. Figure 9 - Cumulative Pumping Volume is a plot of time vs. pumping volume superimposed on the hydrograph curve of Figure 2 (See Appendix B for the Table of Calculations). This provides a graphic representation of Q_{in} vs. Q_{out} at the pump station site for a 100-year storm. It also shows how frequently pumps will cycle on and off for that magnitude of storm.

Pumps will cycle on:	Pumps will cycle off:
1st - 4953.5 ft.	1st - 4956.0
2nd - 4955.0 ft.	2nd - 4955.0
3rd - 4956.5 ft.	3rd - 4953.5
4th - 4958.0 ft.	4th - 4951.0

VALVE PIT AND HEADER PIPE

Flow from the wet well will be transmitted to the force main through a series of valves (see Pump Station Plan View - Figure 10). An open $1^{\pm}$ in. line bleeding back into the wet well will relieve air in the system and thereby decrease head losses. A swing check valve shall be installed with Victaulic couplings on either side to provide convenient access for valve maintenance. A manually operated butterfly valve shall be placed adjacent to, and serve as back-up for, the check valve. Flows from the large pumps will be collected in a concrete lined header pipe located outside the valve pit. Cathodic protection in the form of a sacrificial anode shall be provided up to the junction of the concrete lined steel pipe with the RCP.

Valve sequencing for the 20 hp pumps is identical to the 214 hp; however, the 10 in. header is located inside the valve pit.

A 4 in. line will be installed from the bottom of the large header pipe to the wet well. Through a series of bends, valves and tees it will also connect to the 10 in. header (see Figures 3 and 10). This line will perform three functions:

- Draining the large header
- Draining the small header (either simultaneously with, or separate from, the large header)

- Provide a detour route for the flow if the 10 in. force main is taken out of service.

FORCE MAIN AND OUTLET CHANNEL

A 48 in. RCP force main and a 10 in. PVC force main will transmit flows each approximately 2,900 L.F. For purposes of calculations, the last 150 L.F. are considered gravity line. Clean-out manhole tees to provide access to the force mains will be installed at 500 ft. intervals.

A riprap-lined channel will convey flow from the force main to the river approximately 170 ft. The irregularity of the lining will dissipate energy during periods of heavy flow.

SURGE CONTROL

Consideration has been given to the type and extent of surge relief device this system will require. A specific determination has not yet been made, however, the Consultant is still investigating. The cost of a surge control tank has been included in the overall cost of the pump station.

STRUCTURAL DESIGN

Functional Units

The pump station structure is made up of three functional units: the inlet channel structure, the wet well and the valve pit. All three are tied together structurally with a reinforced concrete mat foundation and walls. The flowline of the inlet channel is approximately 20 ft. below grade. This structure consists of the channel transition from the outfall of the RCP storm sewer to the outfall to the wet well. The barscreen is supported in the sidewalls of the channel transition. Two overflow ports run adjacent to the channel transition. The wet well structure is approximately 28 ft. deep with an open well area of 36 ft. by 25 ft. The wet well will have an 8 ft. high baffle wall that will dissipate the nap from the inflow structure. A 3 ft. deep sump pit lies in the bottom of the well.

The valve pit is 10 ft. deep and will be covered by a reinforced concrete slab with access ports for servicing the valves. A concrete masonry wall with stucco veneer, 4 to 7 ft. high, will sit on the perimeter concrete walls as a screen wall and will have access gates to the pump well.

Design Considerations

Design considerations for the pump station structure were based on the soils data provided by the geotechnical consultant. Groundwater is expected to be encountered at a depth of 14 ft. As a result, the inlet and wet well structures will experience buoyancy forces with uplift pressures of approximately 500 psf and 1,000 psf, respectively. The total dead weight of the structure must be greater than the total uplift force on the mat foundation. The structure as designed has a factor of safety of 1.3 to uplift. (This includes an additional 2 ft. of pressure head over the recommendations of the soils study.) A continuous bentonite waterproofing applied to the underside of the foundation mat and exterior wall below grade will insure that infiltration will not occur. Lateral pressures for the design of the wall below grade were based on equivalent fluid pressures of 50 psf per foot depth for drained compacted backfill, 85 psf per foot depth for saturated backfill, and a 100 psf surcharge load.

The entire pump station structure was designed as a continuous plate design using the ACI 318-83 Code.

Construction of the wet well and inlet structure will entail an extensive dewatering operation, probably utilizing an open excavation with sump pumps being operated continuously. A total of 522 cubic yards of concrete will be poured to construct the well.

CONTROL BUILDING AND STANDBY POWER

General

The following codes and guides will apply to project design:

- NFPA70-1984 (National Electrical Code)
- Public Service Company of New Mexico (PNM) - Power Planning Guide
- Guide Specifications - SECTION 15972 Programmable Controller Based Control System (October 3, 1986) with changes dated March 17, 1986.

Functional Requirements

Lighting will consist of the following types:

- Non-obtrusive ground level area lighting, high-pressure sodium
- Vaportight fluorescent in valve pit
- Incandescent floods for pump station barscreen and wet well area task lighting
- Fluorescent industrial fixture for control building interior
- Wall pack area lighting as applicable on control building exterior and pump station above grade structure

Photo-electric control will be input to a programmable controller (PC) for either programmed or photo-cell control of lighting. Bi-stable contactor will be used to enable pulse control from the PC.

A Power Source will consist of a preferred and an alternate 12.47 kV primary supply from PNM, with automatic transfer. Figure 11 - Electric One-Line generally describes the anticipated electrical service arrangement.

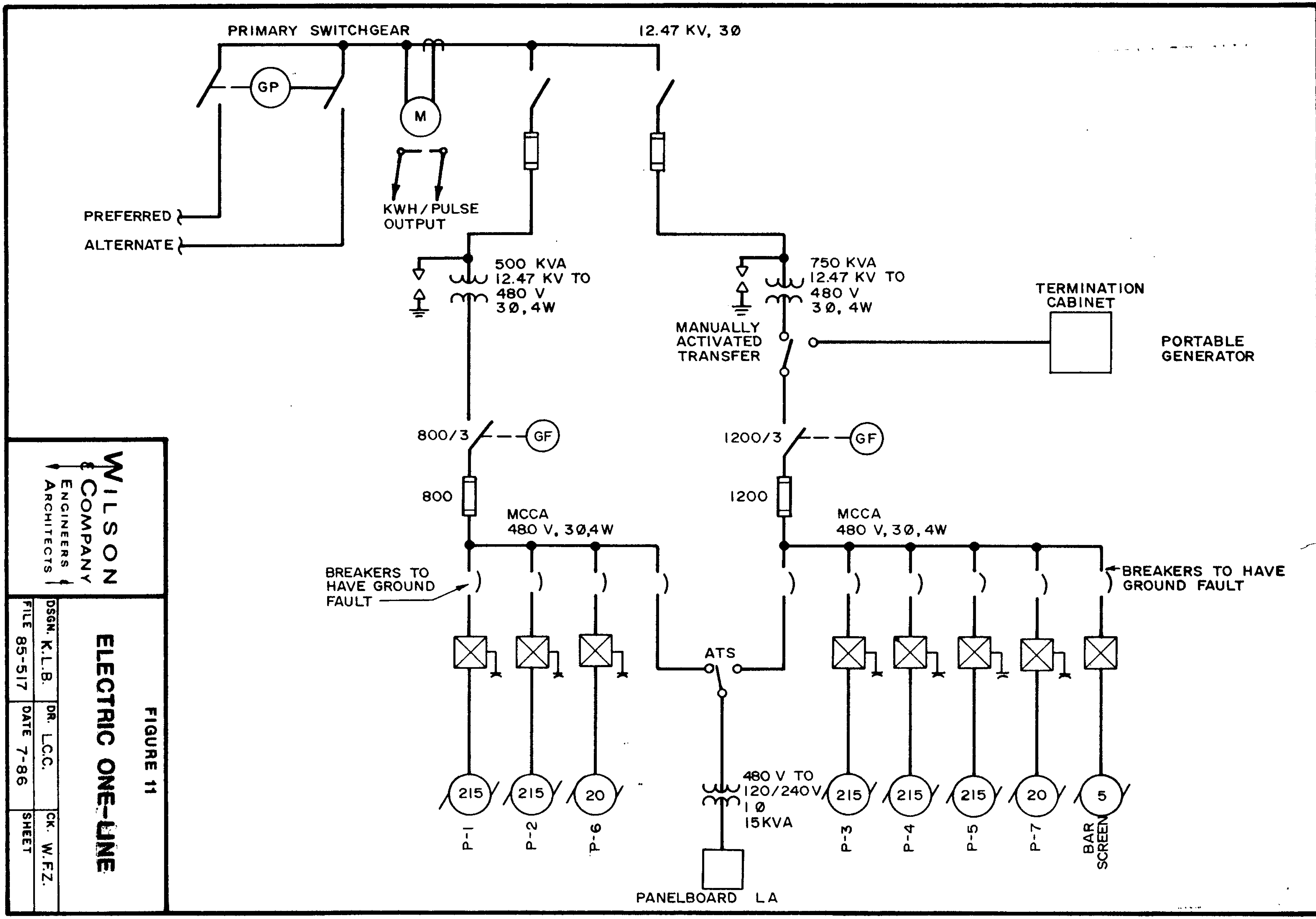

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FIGURE 11

ELECTRIC ONE-LINE

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A contact shall be made available in kWh meter for input of electric consumption to the PC.

A manually initiated transfer switch will provide means to connect to the City's portable emergency generator.

An automatic transfer switch will be employed to transfer station critical loads such as PC, lighting and control power requirements.

Ventilation and Heating will be provided inside the control building only. Control building ventilation will be sized to remove heat due to electrical equipment and due to solar gain.

No special hazardous areas exist at the pump station, thus no ventilation or smoke detection is required.

Pump Control logic shall reside in the PC together with control for lighting, station security, and other functions described by the referenced City's guide specifications. Pumps consist of five main pumps plus two sump pumps. All pumps shall be sequential start. One main pump shall be designated as standby, with a programmed sequence to rotate the designated standby pump into service. Sump pump control shall alternate lead and lag pump. A backspin timer and restart sequence will be resident in the PC software.

Pump start/stop control will be via two admittance probe level sensors in the wet well. An ultrasonic detector will not be used for level control. A high-level alarm for input to telemetry will be provided via a float switch.

It may be desired to require some hardwired control functions to protect pumps during manual (local) operation. The Consultant will evaluate this requirement during final design.

The following pump conditions will be input to the PC:

- Check valve position
- Starter auxiliary contact
- H-O-A selector switch position
- Pump or motor abnormal conditions

The barscreen cleaning rake will be operated when main pumps are operating, except the float switch upstream shall serve the dual purpose of starting barscreen at high level and initiating an alarm via PC. A return to normal level will shut barscreen down, but the alarm will remain on until it is reset.

Intrusion Detection and Station Security Safeguard will be developed as a PC software routine. Control building doors and valve pit hatches will have switch inputs to PC for programmed logic control and telemetered alarm.

It is requested that the City identify station alarm functions (horn, flashing lights, etc.) that would apply to the Montano Pump Station. HID light sources have lag time starting and will not flash.

CONSTRUCTION SEQUENCE

Design Concept

The pump station and RCP storm drain will be constructed during the roadway improvement portion of the project. During construction of the Montano River Crossing the creation of a sump near the Montano/Rio Grande Intersection will require the use of inlets and a 30 in. storm drain to transmit runoff to the pump station. The 30 in. line will be brought into the wet well through a connection made in the side-wall of the inlet channel (see Figure 1). Since the maximum elevation of the wet well is less than top of curb at the inlets, it will not be necessary to flapgate the entrance of the 30 in. line to the inlet channel.

Traffic Control

During the construction of the Rio Grande Boulevard Bridge an alternative travel route through that intersection will be required. The pump station site will provide the necessary detour area with some portions temporarily fenced off for safety. After the bridge is finished, the site will be enclosed with permanent security fencing and final grading, paving and landscaping completed.

ENGINEER'S
DRAINAGE PLAN
&
GRADING PLAN

MONTANO PUMP STATION
PROJECT 3247

LOS POBLANOS RANCH
TRACT 6A
AT
NE CORNER MONTANO ROAD & RIO GRANDE BLVD.
ZONE MAP F-13

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AUGUST 1987
(85-517)

WILSON
& COMPANY
ENGINEERS
ARCHITECTS

ENGINEER'S
DRAINAGE PLAN
&
GRADING PLAN
FOR
MONTANO PUMP STATION
PROJECT 3247

PURPOSE AND SCOPE

The City of Albuquerque plans to construct a storm water pump station at the northeast corner of Montano Road and Rio Grande Blvd. The pump station is a part of the Montano Road and Bridge project and will collect storm water from the storm drain in Montano Road and pump the storm water to the Rio Grande.

SITE LOCATION AND TOPOGRAPHY

The site is a 0.92 acre tract of land located at the northeast corner of Montano Road and Rio Grande Blvd. At the present time the site is developed as a service station.

The slope of the existing ground is virtually flat with slopes of 0.20% or less.

The soils are classified as SCS Hydrologic Soil Type B.

DESIGN CRITERIA

Peak flow volumes for the probable 100-year storm have been determined in accordance with Part C.2, page 7 and Part F, page 19, Chapter 22.2 of Part 2 Development Process Manual published by the City of Albuquerque.

OFF-SITE DRAINAGE

The surrounding off-site lands are flat and are cultivated fields, thus there is a minimal amount of storm water runoff.

The site will be protected by a 6 to 12 inch high dike. The runoff from Rio Grande Blvd. and Montano Road will be collected by the proposed storm drainage improvements.

ON-SITE DRAINAGE

The on-site drainage will be collected by proposed on-site catch basins and then discharged to the wet well of the pump station and then pumped to the Rio Grande.

The attached grading plan shows the catch basins and the on-site drainage areas.

The north drainage area is 0.41 acre and has a peak run-off of 2.42 cfs.

The south drainage area is 0.51 acre and has a peak run-off of 3.02 cfs.

The run-off calculations are attached as a part of this report.

RECOMMENDATION

It is recommended on-site drainage discharge to wet well of the proposed pump station.

HYDROGRAPH COMPUTATION WORKSHEET

DATE 3 Aug 87
COMPUTED BY RFS
CHECK BY ______

PROJECT Montano Pump Station

LOCATION NE Cor Montano & Rio Grande Blvd

ANALYSIS POINT # Catch Basin No 1

(DR. AREA) A = 0.01 ACRES

T_c 10 MIN

POINT RAINFALL 2.2 IN. FROM PLATE 22.2 D-1

CN = 90 FROM PLATES 22.2 C-2, 22.2 C-3

RUNOFF VOLUME R = 1.30 IN. FROM PLATE 22.2 C-4

COMPUTED T_p = 10 MIN. $T_p = T_c$ (Rounded to even minute)

$q_p = \frac{45.4A}{T_p}$ = 1.86 CFS./INCH OF RUNOFF

$(R \times q_p) = Q_{peak}$ = 2.42 CFS

$t(\text{COLUMN}) = (t/T_p)$ $t = T_p(t/T_p)$

$y = \frac{Q}{Q_{peak}}$ $Q = y(Q_{peak})$

	(t/T_p)	t (min.)	y	Q (cfs)
1	0	0	0	0
2	.1		.03	
3	.2		.10	
4	.3		.190	
5	.4		.310	
6	.5		.470	
7	.6		.660	
8	.7		.820	
9	.8		.930	
10	.9		.990	
11	1.0		1.00	
12	1.1		.990	
13	1.2		.930	
14	1.3		.860	
15	1.4		.780	
16	1.5		.680	
17	1.6		.560	
18	1.7		.460	
19	1.8		.390	
20	1.9		.330	
21	2.0		.280	
22	2.2		.207	
23	2.4		.147	
24	2.6		.107	
25	2.8		.077	
26	3.0		.055	
27	3.2		.040	
28	3.4		.029	
29	3.6		.021	
30	3.8		.015	
31	4.0		.011	
32	4.5		.005	
33	5.0		.000	

CN 75 @ 65% Impervious = 90

HYDROGRAPH COMPUTATION WORKSHEET

DATE 3 Aug 87
COMPUTED BY RFS
CHECK BY

PROJECT Montano Pump Station

LOCATION NE Cor Montano & Rio Grande Blvd

ANALYSIS POINT # Catch Basin No 2

(DR. AREA) A = 0.51 ACRES

T_c 10 MIN

POINT RAINFALL 2.2 IN. FROM PLATE 22.2 D-1

CN = 90 FROM PLATES 22.2 C-2, 22.2 C-3

RUNOFF VOLUME R = 1.30 IN. FROM PLATE 22.2 C-4

COMPUTED T_p = 10 MIN. $T_p = T_c$ (Rounded to even minute)

$q_p = \frac{45.4A}{T_p}$ = 2.32 CFS./INCH OF RUNOFF

$(R \times q_p) = Q_{peak}$ = 3.02 CFS

t(COLUMN)=(t/T_p) $t = T_p(t/T_p)$

$y = \frac{Q}{Q_{peak}}$ $Q = y(Q_{peak})$

CN 75 @ 65% Impervious = 90

	(t/T_p)	t (min.)	y	Q (cfs)
1	0	0	0	0
2	.1		.03	
3	.2		.10	
4	.3		.190	
5	.4		.310	
6	.5		.470	
7	.6		.660	
8	.7		.820	
9	.8		.930	
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15	1.4		.780	
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29	3.6		.021	
30	3.8		.015	
31	4.0		.011	
32	4.5		.005	
33	5.0		.000	